GEOTECHNICAL INVESTIGATION

PROVO CITY MUNICIPAL AIRPORT

ATCT NEW EQUIPMENT BUILDING AND RTR ANTENNA FOUNDATIONS

Provo, Utah

Prepared for:
Federal Aviation Administration

April 2011

RB&G
ENGINEERING, INC.

201101-017

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April 29, 2011

Alan W. Smith
FAA Northwest Mountain Regional Office
1601 Lind Ave., SW
Renton, WA 98057

Re: Provo City Municipal Airport
ATCT New Equipment Building and RTR Antenna Foundations

Dear Mr. Smith:

A Geotechnical Investigation has been completed for the proposed ATCT New Equipment Building and RTR Antenna Foundations to be located at the Provo City Municipal Airport in Provo, Utah. The results of the study are summarized in the report transmitted herewith.

We appreciate the opportunity of providing this service for you. If there are any questions relating to the information contained herein, please call.

Sincerely,

RB&G ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
NO.162291
BRADFORD E. PRICE
4/30/11
STATE OF UTAH

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And RTR Antenna Foundations
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Geotechnical Investigation

INTRODUCTION

This report outlines the results of a geotechnical investigation performed for the proposed ATCT New Equipment Building and RTR Antenna Foundations to be located south of the Air Traffic Control Tower at the Provo Municipal Airport in Provo, Utah.

We understand that the new equipment building will be a steel pre-fabricated structure approximately 20 feet wide and 22 feet long with a weight of about 20 kips. Equipment racks with loads up to 20 kips, and a 5 kip battery rack will result in floor loading of about 70 psf. We also understand that the two identical RTR tilt-down antennas will be supported using 36 inch diameter drilled pier foundations.

RB&G Engineering performed a geotechnical investigation for the Air Traffic Control Tower in August 2003, and information obtained during that study will be utilized in preparation of this report, where applicable.

The purpose of this investigation was to determine the characteristics of the subsurface material throughout the site so that satisfactory substructures can be designed to support the proposed facilities. The results of the investigation, along with pertinent recommendations for foundation design, are outlined in the following sections of this report.

The information contained in the report is discussed under the following headings: (1) Geological and Existing Site Conditions, (2) Field and Laboratory Testing Procedures, (3) Subsurface Soil and Water Conditions, (4) Foundation Considerations and Recommendations, and (5) Site Preparation and Compacted Fill Requirements.

I. GEOLOGICAL AND EXISTING SITE CONDITIONS

The Provo Municipal Airport is located at the extreme southwest end of Provo City adjacent to Utah Lake. The natural soils throughout the area consist predominantly of post Lake Bonneville Lacustrine marsh and alluvial deposits. Granular fill has been placed throughout the airport development to provide a stable subgrade. The Lincoln Point Fault has been identified approximately 3.5 miles west of the airport beneath Utah Lake, and the Wasatch Fault is located approximately 5 miles east of the airport, near the base of the Wasatch Mountain Range. Utah County Natural hazards maps identify this area as having high liquefaction potential.

Granular fill has been used to provide a raised platform for the Air Traffic Control Tower. The granular fill slopes downward several feet from the tower site at a rate of about 10% to the natural ground surface, which is at about elevation 4491 feet.

Utah Lake is located a short distance west of the site, with a levee between the site and the lake. At the time of the field investigation, the lake level was at 4489.5 feet msl. The lake reached an elevation of about 4492 feet during the 1980's.

The groundwater level is influenced by the water level in Utah Lake, seasonal precipitation, and irrigation practices in the general area. The control tower is supported on deep pile foundations extending to about elevation 4395 feet. Most structures at the Provo Airport have been supported using spread foundations on compacted sandy gravel. Foundation performance for buildings in the immediate vicinity of the site appears to have been adequate, in that no apparent cracks were observed in the foundation walls. Other than the information provided above, no conditions appear to exist at this site which would adversely affect foundation performance.

Other than the information provided above, no conditions appear to exist at this site which would adversely affect foundation performance.

II. FIELD AND LABORATORY TESTING PROCEDURES

The subsurface investigation was performed using a CME 55 rotary drill rig with a tri-cone rock bit and NW casing to advance the boring and water as the drilling fluid. During the subsurface investigation, sampling was generally performed at one- to three-foot intervals in the upper 15 feet of the soil profile and at five-foot intervals thereafter. Both disturbed and undisturbed samples were obtained during the field investigations. Disturbed samples were obtained by driving a 2-inch split spoon sampling tube through a distance of 18 inches using a 140-pound weight dropped from a height of 30 inches. The number of blows required to drive the sampling spoon through each 6 inches of penetration is shown on the boring logs. The sum of the last two blow counts, which represents the number of blows to drive the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value, corrected for overburden and hammer energy, provides a good indication of the in-place density of sandy material; however, it only provides an indication of the relative stiffness of the cohesive material, since the penetration resistance of materials of this type is a function of the moisture content. Considerable care must be exercised in interpreting the standard penetration value in gravelly-type soils, particularly where the size of the granular particle exceeds the inside diameter of the sampling spoon. If the spoon can be driven through the full 18 inches with a reasonable core recovery, the standard penetration value provides a good indication of the in-place density of gravelly-type material.

Undisturbed samples were obtained at select locations by pushing a thin-walled sampling tube into the subsurface material using the hydraulic pressure on the drill rig. The location at which the undisturbed samples were obtained is shown on the boring logs.

Miniature vane shear tests, which provide an indication of the undrained shearing strength of cohesive materials, were performed on samples of the clay soil during the field investigations. The results of these tests are shown on the boring logs as the torvane value in tsf.

Each sample obtained in the field was classified in the laboratory according to the Modified Unified Soil Classification System. The symbol designating the soil type according to this system, is presented on the boring logs. A description of the Modified Unified Soil Classification System is presented in the appendix, and the meaning of the various symbols, shown on the logs, can be obtained from this figure.

Laboratory tests performed during this investigation to define the characteristics of the subsurface material throughout the proposed site included in-place dry unit weight, natural moisture content, unconfined compressive strength, Atterberg Limits, mechanical analyses, pH, resistivity, sulfate, chloride, and consolidation tests. Testing was performed following procedures outlined in the American Society for Testing and Materials (ASTM) standards.

III. SUBSURFACE SOIL AND WATER CONDITIONS

The characteristics of the subsurface material were evaluated using the deep boring drilled for the control tower in 2003 (03-1) and by drilling one boring to a depth of 36.5 feet and two borings to 31.5 feet during this investigation at the approximate locations shown in Figure 2. The logs for the borings are presented in the appendix.

It will be noted from the 2011 borings that granular fill was encountered in each of the borings varying in thickness from 2 to 5 feet. The fill is underlain by firm to soft cohesive soils with occasional sand layers extending to depths of between 22 and 29 feet below the ground surface, followed by loose to medium dense silty sand to the bottom of the borings at 31 to 36 feet. The deep boring encountered interbedded layers of lean clay, silty sand and sandy silt with occasional fat clay layers. Dense silty sand was encountered below a depth of about 90 feet.

Groundwater was encountered at a depth of between 6 and 8 feet below the existing ground surface at the time the field investigation was performed (April 2011). The depth correlates to about elevation 4489.5 feet.

The results of classification, density and moisture tests are presented on the boring logs, and the results of all laboratory tests, with exception of the consolidation tests, are summarized in Table 1, Summary of Test Data in the appendix. It will be noted from Table 1 that the cohesive soil had an in-place dry unit weight ranging from 62.1 to 90.1 pcf, a natural moisture content varying from 24.8 to 73.3%. The unconfined compressive strength of the cohesive soil ranged from 480 to 1803 psf, with an average of 989 psf. The lean clay had a liquid limit of 35 to 49 and a plasticity index of 14 to 26. The fat clay and elastic silt had a liquid limit of 62 to 67 and a plasticity index of 31 to 40. The silty sand had 0% gravel size particles, 75 to 79% sand, and 21 to 25% non-plastic silt.

The compressibility characteristics of the subsurface material were evaluated by performing two consolidation tests on samples obtained from Boring 11-3 at depths of 6 to 7.5 feet and 15 to 16.5 feet. The results of these tests are also presented in the appendix.

During performance of the consolidation tests, each sample was permitted to absorb water at the beginning of the test to determine the effect of moisture on the compressibility characteristics of these materials. Expansive soils always experience an increase in void ratio on absorbing water. It will be observed from these tests that no increase in the void ratio occurred as the sample absorbed moisture. It is concluded from the consolidation and classification tests that the subsurface materials at this site do not have expansive characteristics. The cohesive soils, in general, have relatively high compressibility characteristics for load intensities greater than 1tsf.

In order to obtain an indication of the corrosive nature of the subsurface material at this site, resistivity, pH, sulfate, and chloride tests were performed on samples obtained from Boring 11-1 at 0 to 1.5 feet, 11-2 at 6 to 7.5 feet, and Boring 11-3 at 3 to 4.5 feet. Results of pH and resistivity tests tests are presented in Table 1, Summary of test data in the Appendix, and it will be noted that these soils have a pH of 7.9 to 8.5 and a resistivity ranging from 120 to 3800 ohm cm. The chloride and sulfate test results will be submitted in an addendum upon completion. The near surface native soils have low resistivity and are considered to have poor corrosion resistance. Testing performed on surface cohesive layers at other locations at the airport has shown low percentages of water soluble sulfate. It is recommended, however, that Type II cement be used for concrete in contact with the native soils due to its increased resistance to sulfate attack.

IV. FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

A. FOUNDATION TYPES AND BEARING CAPACITIES

As indicated earlier in this report, we understand that the new equipment building will be a steel pre-fabricated structure approximately 20 feet wide and 22 feet long with a weight of about 20 kips. Equipment racks with loads up to 20 kips, and a 5 kip battery rack will result in floor loading of about 70 psf. We also understand that the two identical RTR tilt-down antennas will be supported using 36 inch diameter drilled pier foundations.

1. Equipment Building

It appears that up to 2.5 feet of fill will be required to establish final grade for the new equipment building. If final design results in raising the building site more than 3 feet, it is requested that we be notified so settlement associated with large area loads can be evaluated.

Spread Footings on Compacted Fill

It is our opinion that the equipment building can be supported using spread footings on compacted fill provided that post-earthquake settlement following the design seismic event, as described in a subsequent section, can be tolerated. It is recommended that foundations extend to a depth of 2.5 feet below final grade to provide frost protection. The density of the existing fill appears to vary throughout the site. It is recommended, therefore, that all footing areas be over-excavated to a depth of 2 feet below the footing level. The width of the excavations should extend 1 foot beyond the footing perimeter. The on-site granular fill can be used as structural fill, provided that it is placed in lifts not exceeding 8 inches in thickness and compacted to an in-place unit weight equal to at least 95% of the maximum laboratory density as determined by ASTM D 1557. We recommend that imported fill, if required, consist of relatively well-graded sandy gravel having a maximum size of 3 inches with 5 to 15% passing the No. 200 sieve. The imported fill should meet the compaction requirements outlined above for the on-site material.

If the above recommendations are complied with, footings can be designed using the allowable bearing capacity shown below, except that in no case should the width of any footing be less than 2 feet.

FOOTING WIDTH (ft)	ALLOWABLE BEARING CAPACITY (psf)	
	CONTINUOUS FOOTINGS	SPOT FOOTINGS
2	1600	3200
3	1330	2200
4	1200	1800

If the foundations for the proposed facility are designed in accordance with the recommendations outlined above, the maximum settlement of any footing should not exceed one inch and differential settlement throughout the structure should not exceed 0.5 inch under non-seismic loading.

Deep Foundations

If the structure cannot tolerate the estimated settlement associated with liquefaction during the design seismic event, we recommend supporting the structure on deep foundations. Due to the high groundwater level and potential for artesian conditions, it is recommended that driven piles, in lieu of drilled shafts, be used for deep foundation support. The control tower is supported on deep pile foundations and the pile design for the tower is applicable to this site. The following recommendations were obtained from the 2003 control tower report.

Pile capacity analyses have been completed for axial compression loading. The analyses were performed using the FHWA program SPILE, with no skin resistance assumed through the layers of potential liquefaction. The axial compressive single pile capacities for 12, 14 and 16 inch diameter closed end pipe piles extending to elevation 4395 feet are shown in the following table:

PILE DIAMETER (in)	PILE TIP ELEV. (ft)	APPROX. PILE LENGTH (ft)	SKIN RESISTANCE (kips)	END BEARING (kips)	TOTAL CAPACITY (kips)	ALLOWABLE CAPACITY (kips) USING A FACTOR OF SAFETY OF 3.0
12.75	4395	91	141	80	221	73
14	4395	91	164	97	261	87
16	4395	91	206	126	332	110

It is recommended that the closed end piles be filled with concrete. Pipe with wall thickness thinner than 3/8 inch should not be used due to the driving resistance of the medium dense to dense sand layers. For spacing of at least 3 pile diameters, no compressive group reduction factor is required. Taking the sum of the skin friction on the piles in the group and applying a factor of safety of 3 can approximate the allowable uplift capacity of the pile group. Once the group is determined, this value should be compared to the block action and the lesser value used.

It is recommended that the steel pipe pile thickness be reduced by 1/16 inch during design analyses to account for anticipated corrosion of the steel during the life of the structure.

The test boring identified the soil profile to consist of dense to very dense sand from 4395 feet to 4390 feet. Since the ultimate skin resistance is greater than the allowable capacity, the induced load from end bearing should be relatively small, and settlement of the piles should be less than 0.5 inch.

Floor Slabs

We recommend that at least one foot of granular fill and a free-draining granular layer be placed beneath all floor slabs. The free-draining granular layer should be at least 4 inches thick and should have a maximum size less than 1 inch and not more than 5% passing a 200 sieve. The free-draining material should be densified using at least 4 passes of a smooth drum 5-ton vibratory roller or equivalent. If the above specifications are followed, the granular layer will prevent the accumulation of moisture beneath the floor slab and will also serve adequately as a base beneath the floor slabs. Where moisture sensitive flooring, such as tile flooring systems, is planned, it is recommended that a vapor retarder/barrier be placed directly beneath the concrete floor, in lieu of the free-draining granular layer. It is recommended that the vapor barrier conform to ASTM E 1745 Class A requirements. A subgrade modulus of 150 pci can be used for design.

2. RTR Antenna Foundations

Axial compressive resistance and axial uplift resistance have been determined as a function of depth for 2.0, 2.5, and 3.0 foot diameter drilled shafts. The results are included in Figures 3 and 4. Note that these charts use the AASHTO LRFD approach to evaluate Strength Resistance. If the designers are using the ASD method, a factor of safety of 3 should be applied to the nominal resistance shown on the figures to obtain allowable resistance capacities. It is recommended that a lateral bearing pressure of 100 psf/ft be used for the native soils to evaluate lateral capacity.

It is recommended that temporary casing be used to construct the shafts. The design of rebar and concrete should follow established guidelines. If the foundation recommendations presented above are complied with, the maximum settlement of any shaft should not exceed one inch. If this option is used, it is recommended that inspection of the shaft excavations be made by the soils engineer prior to placement of concrete.

B. SEISMIC CONSIDERATIONS

Due to the liquefiable layers within the soil profile, the site is classified as Site Class F, as per Section 1613 of the 2009 International Building Code. The site is located at latitude 40.2161 North and longitude 111.7267 West. Probabilistic peak ground acceleration (PGA) values are tabulated below:

Probabilistic ground motion values in %g.

	10%PE in 50 yr	5%PE in 50 yr	2%PE in 50 yr
PGA	17.16	26.31	43.81
0.2 sec SA	39.27	60.20	109.91
0.3 sec SA	35.35	55.76	105.44
1.0 sec SA	12.23	20.34	41.23

In addition to the probabilistic seismic evaluation, a site specific deterministic analysis for this site was also performed. The analysis shows the Provo segment of the Wasatch Fault (located about 8 km east of the site) to be capable of generating an earthquake having a magnitude of 7.4 to 7.5, with peak ground accelerations at the site on the order of 0.49g.

Liquefaction of the loose silty sand and sandy silt layers will have an effect on the spectral response acceleration at the 1-Second Period. It is recommended that the site coefficients presented in Table 1613.5.3 (1) of the IBC be increased by 15% over the values presented for Site Class E.

The allowable soil bearing pressure indicated above may be increased by one-third where seismic forces are involved in the structural loads. If the frictional resistance of the footings and floor slabs are used to resist seismic forces, we recommend a coefficient of friction of 0.40 be used to calculate these forces. See Section C below for recommendations related to resistance provided by passive earth pressures.

A liquefaction analysis has been performed for the site assuming a seismic event having an acceleration of 0.33g, which is 2/3's of the MCE, which has a probability of exceedence of 2% in 50 years. The results of the analysis indicate that the silty sand layers between 18.5 and 20.5, 23 and 28, and 33 and 40 feet will liquefy during the design seismic event. A sufficient thickness of non-liquefiable soil exists to prevent a surface rupture during the design event. Liquefaction will result in strain related settlement. Up to 3 inches of settlement has been computed for the seismic event.

The effects of the liquefaction will be partially mitigated by the thickness of non-liquefiable soils beneath the structures; however, it is anticipated that up to ¾ inch of differential settlement may occur across the building site.

C. LATERAL EARTH PRESSURES

It is not anticipated that earth-retaining structures will be required for the proposed facilities. If earth-retaining structures are required, however, and if backfilling is performed using granular material, and if the backfill behind the wall is horizontal, we recommend that the earth pressures be calculated using the following equation, along with the earth pressure coefficient outlined below:

$$P = \tfrac{1}{2}\,\gamma\,K\,H^2$$

Where P = total lateral force on wall, plf
K = earth pressure coefficient
γ = unit weight of soil (125 pcf)
H = height of retained soil against wall

The earth pressure coefficient used in designing the walls will depend upon whether the wall is free to move during backfilling operations, or whether the wall is restrained during backfilling. If the wall is free to move during backfilling operations and the backfill material is granular soil, we recommend an active earth pressure coefficient of 0.30 be used in the above equation to calculate the lateral earth pressures. If the walls are restrained from any movement during backfilling and the backfill material is granular soil, we recommend an at-rest earth pressure coefficient of 0.45 be used to calculate the lateral earth pressure.

The additional active earth pressure due to ground acceleration equal to 2/3's of the MCE may be estimated using a coefficient of 0.18. The seismic ground motion will reduce the available passive resistance. This reduction may be accounted for as an earth pressure acting in the direction opposite the passive resistance, and computed using a coefficient of 0.50. The pressure diagrams for these forces may be roughly approximated as inverted triangles, such that the resultant forces of the seismic components act at heights of approximately 2H/3 above the base of the wall.

For non-yielding walls, the increase in earth pressure corresponding to the seismic event may be estimated using the equation $P_{EQ} = a_h \gamma H^2$, where a_h is a seismic coefficient of 0.32. This force is in addition to the at-rest pressure, and acts at a height of about 0.53H above the base of the wall.

It should be recognized that the pressures calculated by the above equation are earth pressures only and do not include hydrostatic pressures. Where hydrostatic pressures may exist behind a retaining structure, we recommend either the wall be designed to resist hydrostatic pressure, or that a drainage system be placed behind the wall to prevent the development of hydrostatic pressures.

V. SITE PREPARATION AND COMPACTED FILL REQUIREMENTS

The vegetative cover throughout the building site consists of sparse grass and weeds. We recommend that the upper 4 inches be stripped from the area to remove the excess organic matter in the upper portion of the soil profile.

We recommend that imported fill used to establish final grade throughout the site consist of granular soil having a maximum size of 4 inches with less than 30% passing a No. 200 sieve. We recommend that the material passing a No. 200 sieve have a plasticity index less than 6. The fill should be compacted to an in-place density equal to at least 92% of the maximum density as determined by ASTM D 1557. Structural fill beneath foundations should meet requirements outlined in Section IV.A.

Grading around the structure should be performed in such a manner that all surface water will flow freely from the area and that no ponding will occur adjacent to the structure which will permit deep percolation into the foundation area. Roof drains should extend well beyond the building lines to prevent seepage into the foundation soils.

Backfilling around foundation walls should be performed using granular material densified to an in-place unit weight equal to at least 90% of the maximum laboratory density indicated above.

VI. LIMITATIONS

The conclusions and recommendations presented in this report are based upon the results of the field and laboratory tests which, in our opinion, define the characteristics of the subsurface material throughout the site in a satisfactory manner. It should be recognized that soil materials are inherently heterogeneous and that conditions may exist throughout this site which could not be defined during this investigation. Since the bearing capacity for foundation design is dependent upon adequate compaction of imported fill, it is requested that testing of the fill be performed under the direct supervision of the soils engineer.

If during construction, conditions are encountered which appear to be different than those presented in this report, it is requested that we be advised in order that appropriate action may be taken.

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Figures

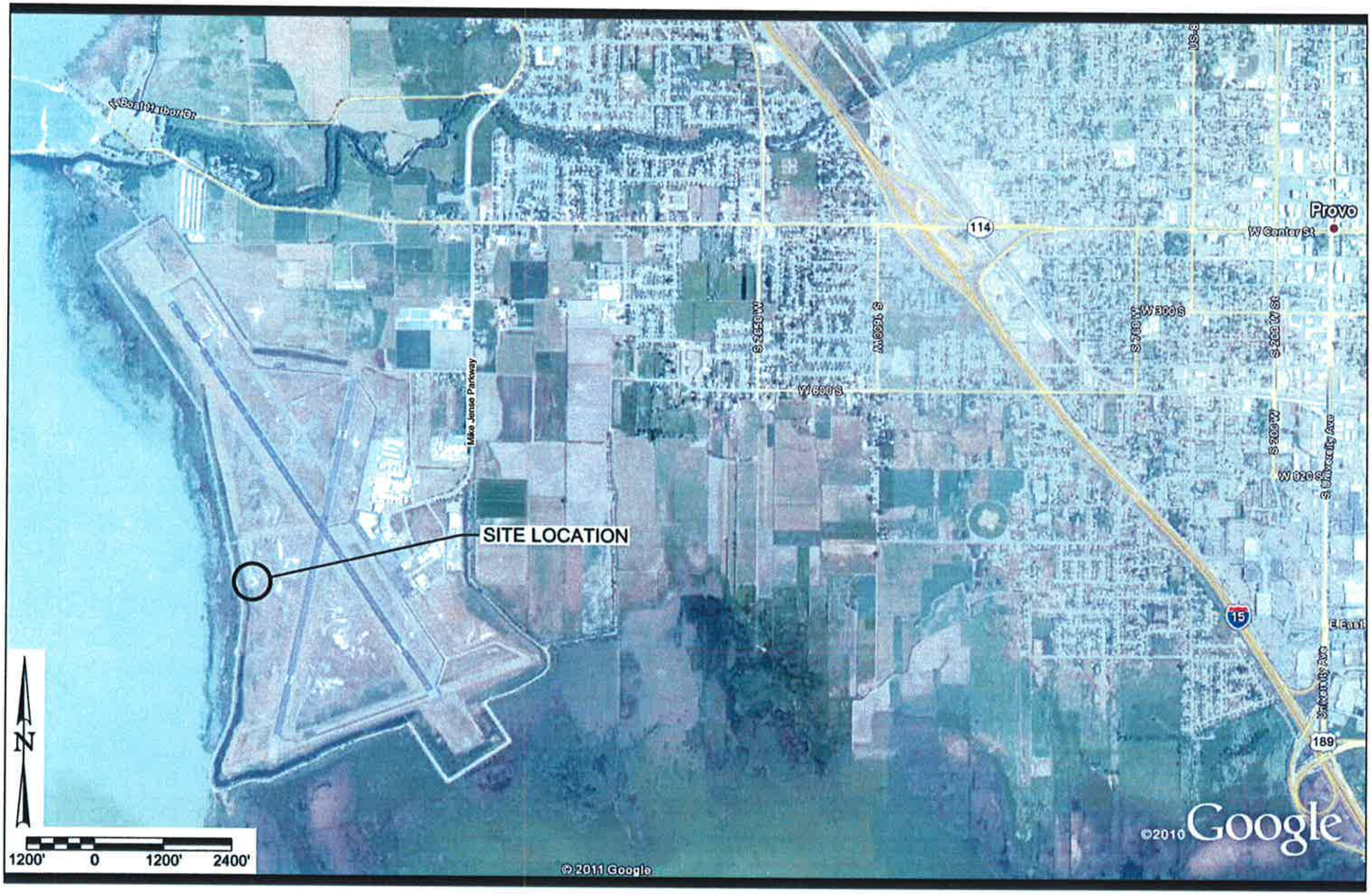

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Figure 1 **VICINITY MAP**

Provo City Municipal Airport ATCT New Equipment Building & RTR Antenna Foundations

Provo, Utah County, Utah

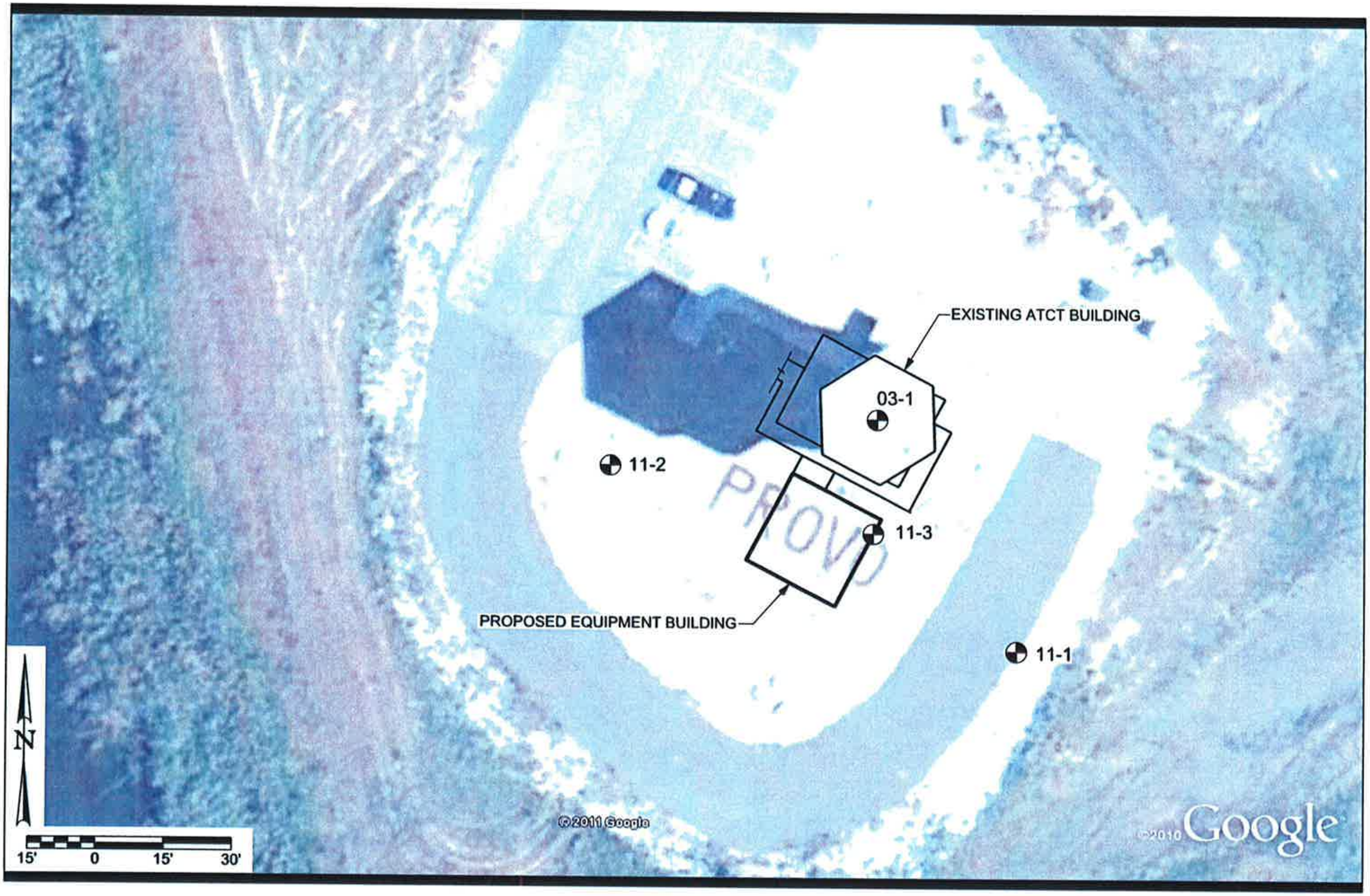

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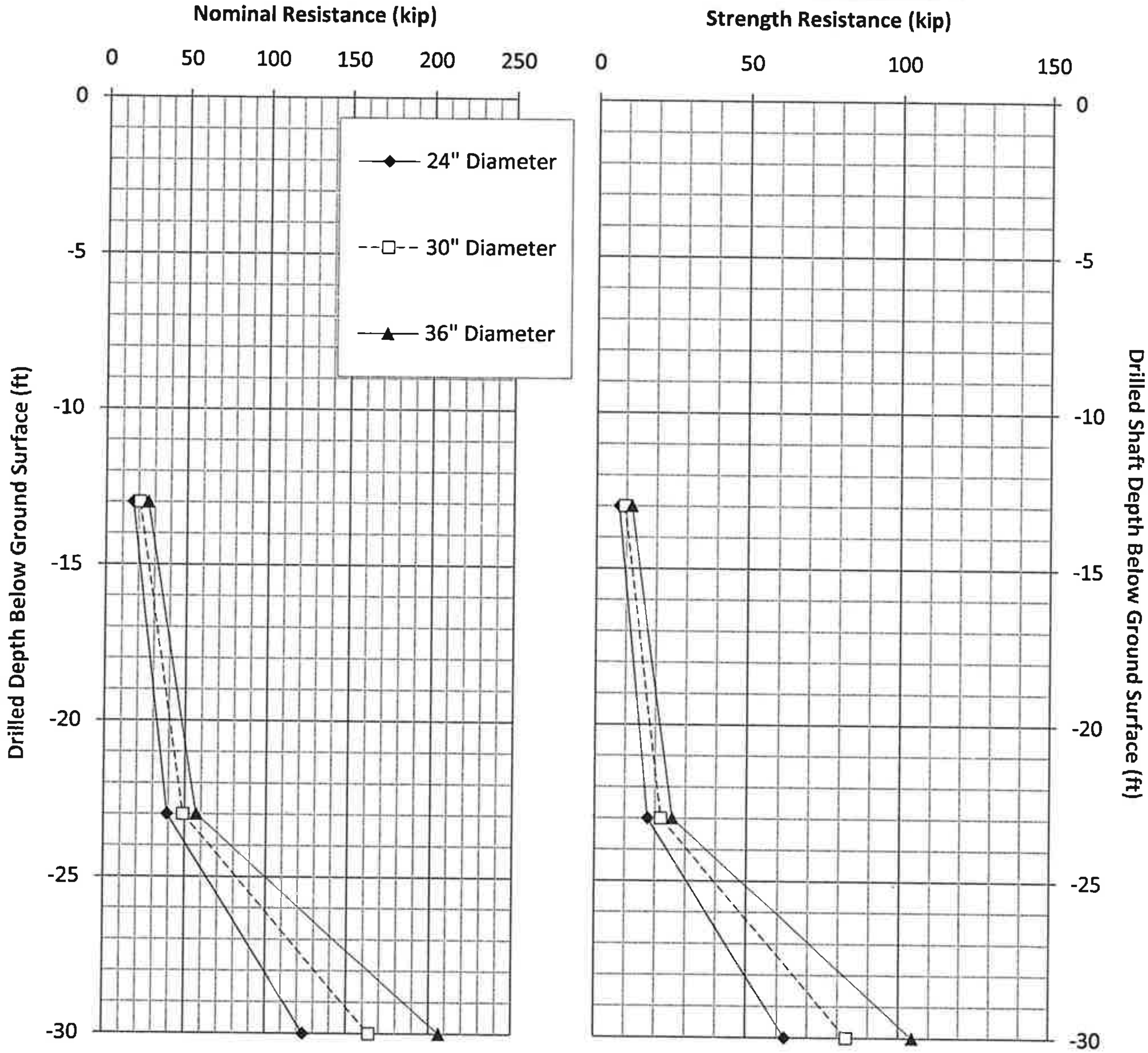
Figure 2 SITE PLAN & TEST HOLE LOCATIONS

Provo City Municipal Airport ATCT New Equipment Building & RTR Antenna Foundations

Provo, Utah County, Utah

DRILLED SHAFT AXIAL RESISTANCE SUMMARY
Provo City Mun. Airport ATCT New Equip. Bldg

Toe Depth (ft)	Axial Compression Resistance (kip)					
	24" Diameter		30" Diameter		36" Diameter	
	Nominal	Strength	Nominal	Strength	Nominal	Strength
-13	17	7	21	9	26	12
-23	39	18	49	22	57	26
-30	124	63	164	83	206	104



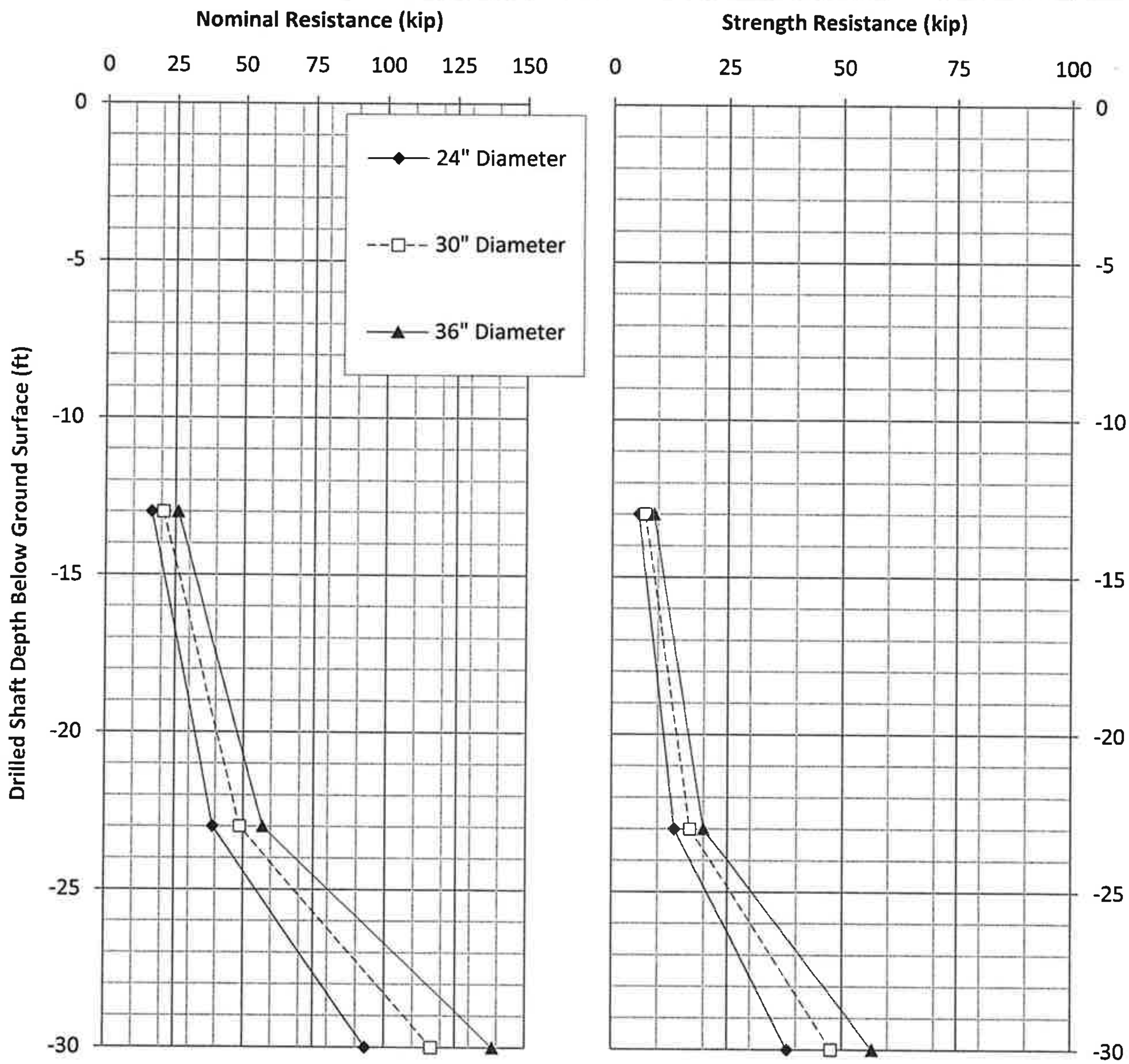
Notes: 1. Reduce these values by 20 percent if shaft is a nonredundant foundation (see AASHTO LRFD 10.5.5.2.4).
2. For shafts spaced less than 4 diameters on centers, apply η factor from AASHTO LRFD 10.8.3.6.3.

FIGURE 3

DRILLED SHAFT UPLIFT RESISTANCE

Provo City Mun. Airport ATCT New Equip. Bldg

Toe Elev (ft)	Axial Uplift Resistance (kip)					
	24" Diameter		30" Diameter		36" Diameter	
	Nominal	Strength	Nominal	Strength	Nominal	Strength
-13	17	6	21	7	26	9
-23	39	14	49	17	57	20
-30	93	38	117	48	138	57



Notes:
1. For extreme event uplift resistance, multiply nominal uplift resistance by resistance factor of 0.80.
2. Further reduce factored resistance by 20 percent if shaft is a nonredundant foundation.
3. Group uplift resistance for shafts spaced at less than four diameters on centers should be evaluated by the geotechnical engineer on a case-by-case basis.

FIGURE 4

Appendix

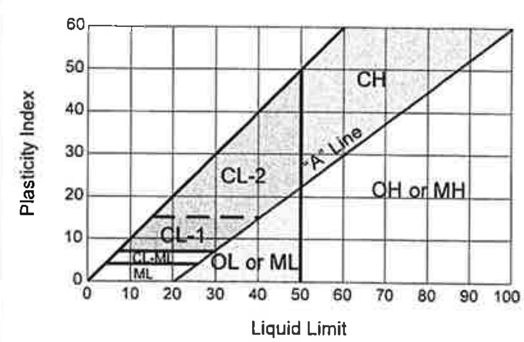
Unified Soil Classification System

Major Divisions			Group Symbols		Typical Names
COARSE-GRAINED SOILS *more than half of material is larger than No. 200 sieve*	**Gravels** *more than half of coarse fraction is larger than No. 4 sieve size*	**Clean Gravels** *little or no fines*	**GW**		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels With Fines *appreciable amount of fines*	**GM***	**d**	Silty gravels, poorly graded gravel-sand-silt mixtures
				u	
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures
	Sands *more than half of coarse fraction is smaller than No. 4 sieve size*	**Clean Sands** *little or no fines*	**SW**		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands, gravelly sands, little or no fines
		Sands with Fines *appreciable amount of fines*	**SM***	**d**	Silty sands, poorly graded sand-silt mixtures
				u	
			SC		Clayey sands, poorly graded sand-clay mixtures
FINE-GRAINED SOILS *more than half of material is smaller than No. 200 sieve*	**Silts and Clays** *liquid limit is less than 50*		**ML**		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	**1**	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				2	
			OL		Organic silts and organic silt-clays of low plasticity
	Silts and Clays *liquid limit is greater than 50*		**MH**		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			**Pt**		Peat and other highly organic soils

Laboratory Classification Criteria

For laboratory classification of coarse-grained soils	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
Determine percentage of gravel and sand from grain-size curve.	Not meeting all gradation requirements for GW	
Depending on percentage of fines (*fraction smaller than No. 200 sieve size*), coarse-grained soils are classified as follows:	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
	Atterberg limits above "A" line, or PI greater	
Less than 5% GW, GP, SW, SP	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
More than 12% GM, GC, SM, SC	Not meeting all gradation requirements for SW	
5% to 12% Borderline cases requiring use of dual symbols**	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
	Atterberg limits above "A" line, or PI greater	

For laboratory classification of fine-grained soils



Plasticity Chart

NOTE: USCS Modified to include CL-type subcategories

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when liquid limit is 28 or less and the PI is 6 or less, the suffix u used when liquid limit is greater than 28.

***Borderline classification*: Soils possessing characteristics of two groups are designated by combinations of group symbols. (For example GW-GC, well graded gravel-sand mixture with clay biner.)

DRILL HOLE LOG

BORING NO. 11-1

PROJECT: PROVO CITY MUN. AIRPORT ATCT NEW EQUIP. BLDG. & RTR ANT. FOUND.

SHEET 1 OF 1

CLIENT: FAA NORTHWEST MOUNTAIN REGIONAL OFFICE

PROJECT NUMBER: 201101.017

LOCATION: SEE SITE PLAN

DATE STARTED: 4/22/11

DRILLING METHOD: 96-CME-55 / N.W. CASING

DATE COMPLETED: 4/22/11

DRILLER: T. KERN

GROUND ELEVATION: ~4496.0'

DEPTH TO WATER - INITIAL: ▽ 6.5' AFTER 24 HOURS: ▼ N.M.

LOGGED BY: J. OLSEN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample Type	Rec. (in)	See Legend	USCS (AASHTO)	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
4495			Disturbed	12	19,19,28,(99)	GM	dk. brown, moist, very dense	2" ASPHALT SILTY GRAVEL W/SAND (fill)								Chem.
			Disturbed	3	8,6,4,(21)	GC	green-brown, very moist, loose	CLAYEY GRAVEL W/SAND (fill)								
4490 ▽	5		Pushed	10	Pushed 0.45	MH	black, moist, firm	ELASTIC SILT	69.7	44.3	63	31				UC 1803 psf
4485	10		Pushed	14	Pushed 0.24	CL-2	black, very moist, soft		79.9	42.1	49	26				UC 910 psf
			Disturbed	13	0/18",(0) 0.10	CL	dk. brown, very moist, very soft	LEAN CLAY W/SAND silt layers, shells, sand layers to 0.5" thick								
4480	15		Disturbed	18	0/12",2,(3) 0.20	CL	dk. brown, very moist, soft									
4475	20		Pushed	11	Pushed 0.16	CH	gray, very moist, soft	FAT CLAY W/SAND shells	62.1	73.3	62	36				UC 638 psf
4470	25		Disturbed	16	1,1,2,(4)	SM	gray, wet, very loose	SILTY SAND clay layers to 0.5" thick								
4465	30		Disturbed	15	6,8,11,(25)	SM	gray, wet, med. dense	SILTY SAND								
4460	35		Disturbed	15	5,7,10,(21)	SM	gray, wet, med. dense									
								BOH								

DH_LOGV1 017_PMAIRPORT.GPJ US EVAL.GDT 4/30/11

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE: 2,3,2,(6) — Blow Count per 6", (N_1)$_{60}$ Value; 0.45 — Torvane (tsf)

UNDISTURBED SAMPLE: PUSHED 0.45 — Torvane (tsf)

OTHER TESTS
- UC = Unconfined Compression
- CT = Consolidation
- DS = Direct Shear
- UU = Unconsolidated Undrained Triaxial
- CU = Consolidated Undrained Triaxial
- HYD = Hydrometer
- SS = Soluble Salt
- DC = Dispersive Clay
- Chem. = pH, Resistivity, Sulfate, Chloride

DRILL HOLE LOG

BORING NO. 11-2

SHEET 1 OF 1

PROJECT: PROVO CITY MUN. AIRPORT ATCT NEW EQUIP. BLDG. & RTR ANT. FOUND.
CLIENT: FAA NORTHWEST MOUNTAIN REGIONAL OFFICE
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ 6.0' **AFTER 24 HOURS:** N.M.

PROJECT NUMBER: 201101.017
DATE STARTED: 4/22/11
DATE COMPLETED: 4/22/11
GROUND ELEVATION: ~4495.0'
LOGGED BY: J. OLSEN, J. BOONE

Elev. (ft)	Depth (ft)	Rec. (in)	See Legend	USCS (AASHTO)	Material Description		Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
		13	7,8,20,(59)	GC-GM	brown, very moist, dense	SILTY CLAYEY GRAVEL W/SAND (fill)								
		9	9,7,6,(27) 0.80	SM	black, moist	SILTY SAND organics								
				CL	dk. brown, moist, stiff	LEAN CLAY W/SAND organics, silt & sand layers to 1" thick								
4490	5													
		17	Pushed 0.12	CL-1	brown, wet, soft			36.1	36	14				Chem.
		18	1,0,2,(4) 0.16	CL	brown, wet, soft									
				CL	brown, wet									
4485	10	16	Pushed	SM	brown, wet	SILTY SAND		28.0		NP	0	79	21	
		11	2,1,2,(5)	SM/ML	brown, wet, loose	SILTY SAND TO SANDY SILT								
		18	1,0/12",(0) 0.20	SM/ML	brown, wet									
				MH	dk. brown, very moist, soft	ELASTIC SILT								
4480	15	18	Pushed 0.35	CH	gray, very moist, firm	FAT CLAY W/SAND organics, shells								
4475	20	18	0/18",(0) 0.30	CH	gray, very moist, firm									
4470	25	16	Pushed	SM	gray, wet	SILTY SAND		34.6		NP	0	75	25	
		14	0,2,4,(8)	SM	gray, wet, loose									
4465	30	13	6,12,15,(36)	SM	gray, wet, dense									
						BOH								

DH_LOGV1 017_PMAIRPORT.GPJ US EVAL.GDT 4/30/11

RB&G ENGINEERING, INC.

LEGEND:
DISTURBED SAMPLE — 2,3,2,(6) Blow Count per 6" / $(N_1)_{60}$ Value; 0.45 Torvane (tsf)
UNDISTURBED SAMPLE — PUSHED; 0.45 Torvane (tsf)

OTHER TESTS
UC = Unconfined Compression
CT = Consolidation
DS = Direct Shear
UU = Unconsolidated Undrained Triaxial
CU = Consolidated Undrained Triaxial
HYD = Hydrometer
SS = Soluble Salt
DC = Dispersive Clay
Chem. = pH, Resistivity, Sulfate, Chloride

DRILL HOLE LOG

BORING NO. 11-3

SHEET 1 OF 1

PROJECT: PROVO CITY MUN. AIRPORT ATCT NEW EQUIP. BLDG. & RTR ANT. FOUND.
CLIENT: FAA NORTHWEST MOUNTAIN REGIONAL OFFICE
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ 8.0' AFTER 24 HOURS: ▼ N.M.

PROJECT NUMBER: 201101.017
DATE STARTED: 4/22/11
DATE COMPLETED: 4/22/11
GROUND ELEVATION: ~4497.5'
LOGGED BY: J. OLSEN, J. BOONE

Elev. (ft)	Depth (ft)	Sample Rec. (in)	See Legend	USCS (AASHTO)	Material Description		Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
		10	6,9,8,(36)	GC-GM	brown, very moist, med. dense	SILTY CLAYEY GRAVEL W/SAND (fill)								
4495		12	13,24,22,(97)	GC-GM	brown, very moist, very dense									Chem.
	5	8	Pushed 0.35	CL-1	dk. brown, moist, firm	LEAN CLAY W/SAND elastic silt layers to 3" thick, sand layers to 4" thick	90.1	24.8	35	14				CT UC 973 psf
4490		2	Pushed	CL	dk. brown, wet									
	10	15	1,2,2,(7) 0.22	CL	brown, very moist, soft									
4485		18	0/12",1,(2) 0.15	CL	dk. brown, very moist, soft									
	15	18	Pushed 0.25	CL-2	brown, very moist, soft to firm		74.8	40.1	48	22				CT UC 1061 psf
4480		13	0,2,3,(8) 0.25	CL	gray, very moist, soft to firm									
	20	17	1,0,1,(1) 0.25	SM	dk. gray, wet, very loose	SILTY SAND								
				CH	gray, very moist to wet, firm	FAT CLAY W/SAND organics, shells								
4475	25	13	Pushed 0.25	CH	gray, wet, soft		63.8	64.5	67	40				UC 1060 psf
4470	30	12	5,8,12,(26)	SM	gray, wet, med. dense	SILTY SAND								
4465						BOH								

DH_LOGV1_017_PMAIRPORT.GPJ US EVAL.GDT 4/30/11

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE: 2,3,2,(6) 0.45 — Blow Count per 6"; $(N_1)_{60}$ Value; Torvane (tsf)

UNDISTURBED SAMPLE: PUSHED 0.45 — Torvane (tsf)

OTHER TESTS
UC = Unconfined Compression
CT = Consolidation
DS = Direct Shear
UU = Unconsolidated Undrained Triaxial
CU = Consolidated Undrained Triaxial
HYD = Hydrometer
SS = Soluble Salt
DC = Dispersive Clay
Chem. = pH, Resistivity, Sulfate, Chloride

DRILL HOLE LOG

BORING NO. 1

Sheet: 1 of 2

PROJECT: PROVO CITY MUNICIPAL AIRPORT TOWER
PROJECT NO.: 200301.024
CLIENT: WILLIAM E. PAYNE & ASSOCIATES, INC.
DATE: 8/4/03
LOCATION: SEE SITE PLAN
ELEVATION: ~4491'
DRILLER: D. SAMPSON, A. KESLER
LOGGED BY: M. CALL, V.N.B.
EQUIP./DRILL METHOD: CME-55 / N.W. CASING
DEPTH TO WATER - INITIAL: N.M. AFTER 24 HOURS: 8.9' ON 8/15/03

Elev. (Feet)	Depth (Feet)	SAMPLE Rec. (In.)	SAMPLE Blows Per 6" & Torvane (tsf)	SAMPLE USCS	Material Description		Dry Density, pcf	Moisture Content, %	Atter. Liquid Limit, %	Atter. Plasticity Index, %	Gradation Gravel, %	Gradation Sand, %	Gradation Silt/Clay, %	Other Tests
4490		12	2,3,2	CL	dk. brown, slightly moist	LEAN CLAY W/ROOTS								
		8	Pushed 0.40	CL	brown, moist, firm									
	5	18	Pushed	ML/SM	brown-gray, moist, firm	SILTY SAND TO SANDY SILT								
4485		18	Pushed 0.36	CL	dk. brown, wet, firm	SANDY LEAN CLAY								UC
		18	1,2,1	ML	dk. brown, wet, very loose	SANDY SILT W/SILTY SAND LAYERS 1"-2" THICK		35.5	31	6				
	10	18	Pushed 0.11	ML										
4480		18	1,2,2 0.12	ML SM	dk. brown, wet, soft / dk. brown, wet, very loose	SANDY SILT								
		9	5,8,4	SM	dk. brown, wet, med. dense	SILTY SAND		36.3		NP	0	58	42	
4475	15	18	Pushed 0.61	OH	dk. gray, wet, stiff	ORGANIC CLAY W/CRYSTALIZED MINERAL DEPOSITS	52.1	80.6	99	51				CT UC
4470	20	18	2,1,1 0.25	CL ML	dk. gray, wet, firm / lt. gray, wet, very loose	SANDY SILT W/CLAY LENSES & LAYERS		38.7	34	8				
4465	25	13	Pushed	SP-SM	gray, wet, loose	POORLY GRADED SAND W/SILT								
		9	0,5,7	SP-SM	gray, wet, med. dense			25.7			0	92	8	
4460	30	14	6,10,11	SM	gray, wet, med. dense	SILTY SAND								
4455	35	16	4,3,5	SM	gray, wet, loose	SILTY SAND W/SANDY SILT LENSES & LAYERS, SOME ORGANICS		40.9			0	60	40	
4450	40	18	1,1,2 0.26	CL-2	gray, wet, soft to firm	LEAN CLAY W/SAND LENSES & LAYERS, SHELLS & ORGANICS		41.4	43	24				
4445	45	0	Pushed	-										
		18	Pushed 0.23	CL-2	gray, wet, soft	LEAN CLAY		46.2	44	18				UC
	50													

FIGURE 2

RB&G ENGINEERING INC.
Provo, Utah

LEGEND

2,3,2 ← Blow Count per 6"
0.45 ← Torvane (tsf)
Disturbed Sample
Undisturbed Sample
Groundwater Elevation

UC - Unconfined Compression Test
CT - Consolidation Test
SG - Specific Gravity Test

DRILL HOLE LOG

PROJECT: PROVO CITY MUNICIPAL AIRPORT TOWER
PROJECT NO.: 200301.024
CLIENT: WILLIAM E. PAYNE & ASSOCIATES, INC.
DATE: 8/4/03 TO 8/5/03
LOCATION: SEE SITE PLAN
ELEVATION: ~4491'
DRILLER: D. SAMPSON, A. KESLER
LOGGED BY: M. CALL, V.N.B.
EQUIP./DRILL METHOD: CME-55 / N.W. CASING
DEPTH TO WATER - INITIAL: N.M. AFTER 24 HOURS: 8.9' ON 8/15/03

BORING NO. 1

Sheet: 2 of 2

Elev. (Feet)	Depth (Feet)	Sample Type	Rec. (in.)	Blows Per 6" & Torvane (tsf)	USCS	Material Description		Dry Density, pcf	Moisture Content, %	Atter. Liquid Limit, %	Atter. Plasticity Index, %	Gradation Gravel, %	Gradation Sand, %	Gradation Silt/Clay, %	Other Tests
4440		Disturbed	18	2,2,2 0.23	CL	gray, wet, soft	LEAN CLAY								
4435	55	Undisturbed	18	Pushed 0.14	CH	gray, wet, very soft	FAT CLAY	60.0	66.9	64	36				CT UC
4430	60	Disturbed	18	2,3,3 0.15	CL	gray, wet, soft	LEAN CLAY								
4425	65	Undisturbed	18	Pushed 0.36	CL-2	gray, wet, firm			42.4	48	23				UC
4420	70	Disturbed	18	4,7,6 0.32	CL	gray, wet, firm	SANDY LEAN CLAY W/SAND LENSES & LAYERS								
4415	75	Undisturbed	12	Pushed 0.25	CL-2	gray, moist to wet, soft, 2" sand lense	LEAN CLAY	75.0	46.2	47	22				CT UC
4410	80	Disturbed	18	3,4,3 0.29	CL	gray, moist to wet, soft	LEAN TO FAT CLAY W/SILTY SAND LAYERS TO 1" THICK								
4405	85	Undisturbed	18	Pushed 0.40	CH	dk & lt. gray, moist to wet, firm			53.7	57	32				UC
4400	90	Disturbed	18	5,9,13	SM	dk gray, wet, med. dense	VERY SILTY FINE SAND W/SOME CLAY								
4395	95	Disturbed	14	11,19,28	SM	lt. gray-brown wet, dense			23.0		NP	0	54	46	DS
4390	100	Disturbed	12	17,31,43	SM	dk. gray wet, very dense									

LEGEND

- 2,3,2 — Blow Count per 6"
- 0.45 — Torvane (tsf)
- Disturbed Sample
- Undisturbed Sample
- Groundwater Elevation

RB&G ENGINEERING INC.
Provo, Utah

FIGURE 2A

DS - Direct Shear Test
UC - Unconfined Compression Test
CT - Consolidation Test
SG - Specific Gravity Test

RB&G ENGINEERING, INC.

Table 1

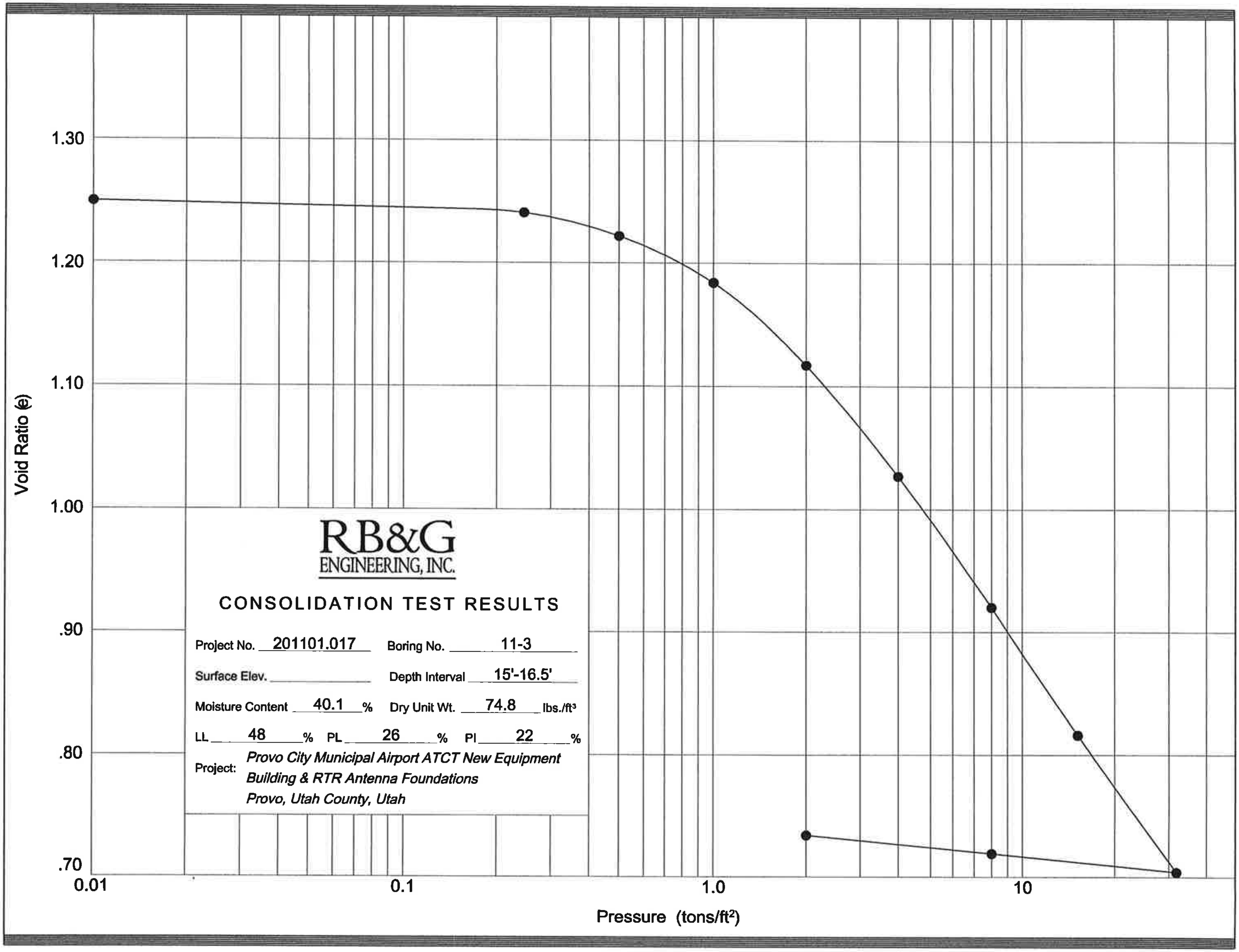
SUMMARY OF TEST DATA

PROJECT	Provo Airport ATCT Bldg.	PROJECT NO.	201101-017
LOCATION	Provo,Ut see site plan	FEATURE	Foundations

HOLE NO.	DEPTH BELOW GROUND SURFACE (ft)	IN-PLACE		UNCONFINED OR UU TRIAXIAL COMPRESSIVE STRENGTH (psf)	ATTERBERG LIMITS			MECHANICAL ANALYSIS			PERCENT FINER THAN 0.005 mm	UNIFIED SOIL CLASSIFICATION SYSTEM / (AASHTO CLASSIFICATION)
		DRY UNIT WEIGHT (pcf)	MOISTURE (%)		LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	PERCENT GRAVEL	PERCENT SAND	PERCENT SILT & CLAY		
11-01	6-7.5	69.7	44.3	1803	63	32	31					MH
	9-10.5	79.9	42.1	910	49	23	26					CL-2
	20-21	62.1	73.3	638	62	26	36					CH
11-02	6-7.5		36.1	480*	36	22	14					CL-1
	9-10.5		28.0				NP	0	79	21		SM
	25-26.5		34.6				NP	0	75	25		SM
11-03	6-7.5	90.1	24.8	973	35	21	14					CL-1
	15-16.5	74.8	40.1	1061	48	26	22					CL-2
	25-26.5	63.8	64.5	1060	67	27	40					CH

		pH	Resistivity ohm cm	Chloride mg/kg-dry	Sulfate mg/kg-dry
11-01	0-1.5	8.5	3800		
11-02	6-7.5	8.0	120		
11-03	3-4.5	7.9	570		

*Torvane value used to estimate unconfined compressive strength.
NP=Non-Plastic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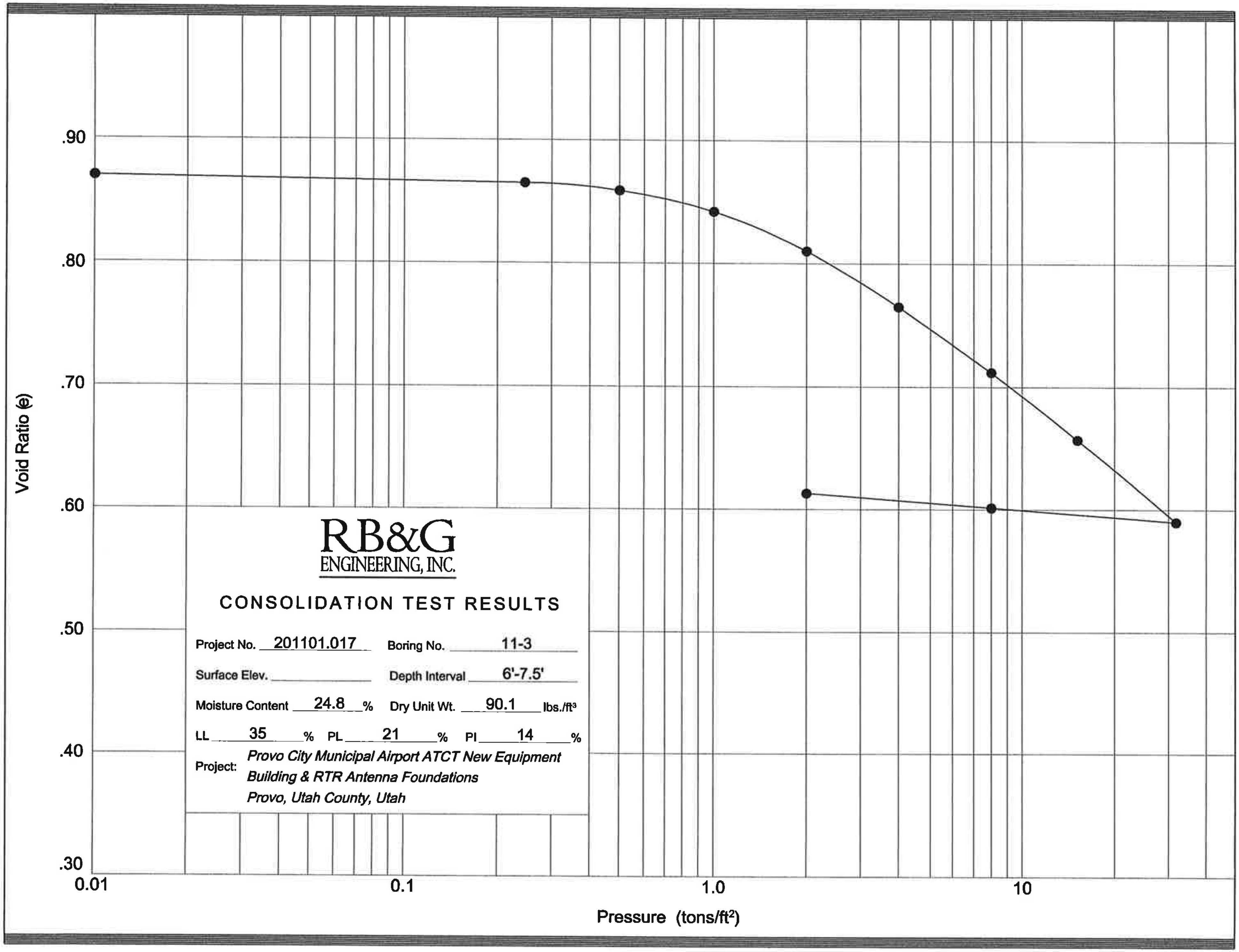
ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Project No. 201101.017
Boring No. 11-3
Surface Elev.
Depth Interval 15'-16.5'
Moisture Content 40.1 %
Dry Unit Wt. 74.8 lbs./ft³
LL 48 %
PL 26 %
PI 22 %
Project: Provo City Municipal Airport ATCT New Equipment Building & RTR Antenna Foundations Provo, Utah County, Utah
Void Ratio (e)
1.30
1.20
1.10
1.00
.90
.80
.70
0.01
0.1
1.0
10
Pressure (tons/ft²)



Void Ratio (e)
.90
.80
.70
.60
.50
.40
.30
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G
ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Project No. 201101.017
Boring No. 11-3
Surface Elev.
Depth Interval 6'-7.5'
Moisture Content 24.8 %
Dry Unit Wt. 90.1 lbs./ft³
LL 35 %
PL 21 %
PI 14 %
Project: Provo City Municipal Airport ATCT New Equipment
Building & RTR Antenna Foundations
Provo, Utah County, Utah